APPENDIX 10-1

CANAL ALTERNATIVES TECHNICAL MEMORANDUM

BLACK & VEATCH

TECHNICAL MEMORANDUM

South Florida Water Management District	B&V Project 141731
EAA Reservoir A-1	B&V File:
Work Order No. 7	First Issue: June 22, 2005
	Last Updated: July 21, 2005

Canal Alternatives Technical Memorandum

To: Distribution

From: Chris Brockway

1. INTRODUCTION

In October 2003, South Florida Water Management District (District) decided to pursue a "Dual Track" for the Everglades Agricultural Area (EAA) Reservoir project. While the multi-agency Project Delivery Team, lead by the Corps of Engineers, continues to develop the Project Implementation Report, the District is proceeding with the design of a reservoir (designated EAA Reservoir A-1 Project) located on land acquired through the Talisman exchange in the Everglades Agricultural Area.

The purpose of the Project as defined in the CERP is to capture EAA Basin runoff and releases from Lake Okeechobee. The facilities will be designed to improve the timing of environmental water supply deliveries to STA 3/4 (Storm Water Treatment Area 3/4) and the WCA's (Wetland Conservation Areas), reduce Lake Okeechobee regulatory releases to the estuaries, meet supplemental agricultural irrigation demands, and increase flood protection within the EAA.

This Canal Technical Memorandum addresses canals associated with the A-1 Reservoir including the North New River Canal, the Northeast Pump Station Connector Canal, the Seepage Canals, the Internal Perimeter Canals, and other associated canals.

2. OBJECTIVES

The objectives of this Section are to:

- Establish capacities and limitations of the existing North New River Canal.
- Identify potential improvements to enhance capacity of the North New River Canal.
- Identify additional canals required to serve the project (Northeast Pump Station Connector, Seepage, and Internal Perimeter).
- Identify impacts to existing canals (agricultural).

3. EXISTING CANAL CAPACITIES AND LIMITATIONS

3.1 North New River Canal

The North New River (NNR) Canal extends from Lake Okeechobee south past the proposed EAA Reservoir A-1 as shown on Figure 1. The canal is intersected by the Bolles and Cross Canals north of the proposed reservoir.

This technical memorandum does not include future capacity needs based on the 2x2 model and ADA pending receipt of finalized information. A.D.A. Engineering is conducting the Everglades Agricultural Area Regional Feasibility Study, which was initiated under the auspices of the State's "Long-Term Plan for Achieving Everglades Water Quality Goals". The focus of the study as it pertains to the EAA Storage Reservoir (EAA SR) project is on re-distributing flows and phosphorus loads to optimize the water quality treatment function of the Everglades Construction Project stormwater treatment areas.. The work is currently in progress and will be completed later in 2005. A.D.A. expects that they will be able to make improvements to the Bolles, Cross and Ocean canals that will allow an additional 5,000 cfs to be delivered to the intersection of the NNR and Cross Canals (Copp, 2005). The amount of additional water available at the intersection of the NNR and Cross Canals will be confirmed at a later date. The amount that will be able to be transported south through the NNR to the site of the A-1 Reservoir, after improvements to the NNR are made, has not yet been determined.

The existing HEC-RAS model for the NNR Canal was used to model the hydraulics of the canal based on various anticipated flows to the proposed Northeast Pump Station and the G-370 Pump Station, flow into the canal from the Bolles and Cross Canals, and local inflow (rainfall) within the drainage area served by the canal. The NNR Canal has the following characteristics:

- Earthen side slopes.
- Irregular profile and cross sections.
- Several "choke" points between Lake Okeechobee and the G-370 pump station located at the southeast corner of the A-1 Reservoir site.
- Flow is currently to and from Lake Okeechobee, the Bolles & Cross canals, and control structure S-7.
- An existing service area for drainage and irrigation.

The modeling of the North New River Canal to determine the available flow at the reservoir is based on the following assumptions:

- Flow rate available at the Bolles & Cross canals is 2,000 to 3,000 cfs (per ADA).
- Current average discharge during wet weather events based on about ¾ inch per acre per 24 hours.
    - 1015 cfs between the Bolles & Cross canals and the Northeast Pump Station.
    - 745 cfs between Northeast Pump Station and G-370.
- 2 feet of freeboard is maintained at all points in the canal.
- Water surface elevation at the intersection with the Bolles & Cross canals cannot exceed 10.6 NAVD 1988.

The profile of the NNR Canal is shown on Figure 2.

The calculated capacity of the canal is based on a Manning's n = 0.030. A discussion of the Manning's n value selected for this analysis will be included in the future BODR under "Headloss". Because of the irregular profile, the available capacity of the canal is entirely dependent on the energy gradient. Therefore, capacity will vary depending on where the flow enters and where it is removed from the canal. For purposes of this analysis, it was assumed that a new pumping station would be constructed at the northeast corner of the A-1 Reservoir.

The current average wet weather discharge of ¾ inch per acre per 24 hours is based on current permitted agricultural drainage pumping capacity. The permitted discharge from the agricultural areas is based on a formula that allows the largest runoff rates for the smaller farms with lesser rates as farms increase in acreage. The least amount allowed by formula is ¾ inch per acre per 24 hours. To put this into perspective, a rainfall event with an expected one year recurrence frequency is 4 inches per 24 hours. Therefore, a ¾ inch per 24 hour and larger event could be expected multiple times each year.

The existing capacities shown in Table 1 were calculated for the canal based on the HEC-RAS modeling with no restrictions on maximum velocity and using the current capacity of Pump Station G-370. As part of the A-1 Reservoir construction, G-370 may be modified and its pumping capacity altered. The actual capacity of G-370 will be established in a future technical memorandum. Available capacities were calculated for both wet periods with rainfall within the NNR Canal drainage area and for dry periods. In addition, two cases were analyzed, the first with all three pumps operating at G-370, the second with one pump out of service. A detailed discussion of the NNR Canal HEC-RAS modeling will be included in a future technical memorandum.

A 1953 Corps of Engineers Report indicates that maximum velocities have been determined as 3.0 feet per second (fps) in areas of sand and other unconsolidated materials and 5.0 fps in areas of rock (Reference 3). In general, canal velocities remain below 3 fps for most of the canal's length for the flow rates shown in Table 1. For the purposes of this analysis, areas where the velocity would exceed 2.5 fps were also identified. Any enhancements identified herein are based on a maximum velocity of 2.5 fps. The modeling indicates that there are areas where the velocity would exceed 2.5 fps as shown in Figure 3. The cross sections referenced in this technical memorandum correspond to the cross sections used in the HEC-RAS modeling of the canal.

3.2 STA 3/4 Inflow Supply Canal

The STA 3/4 Inflow Supply Canal currently conveys water from pump stations G-370 and G-372 to STA 3/4. The canal was designed to convey all flows pumped by the two pumping stations and sized to provide the design feed rate to STA 3/4. No modifications in conveyance capacity are required.

3.3 Miami Canal

Pump Station G-372 currently pumps water from the Miami Canal into the STA 3/4 Inflow Supply Canal. The Miami Canal has greater conveyance capacity than that of the NNR Canal; G-372 has a 3,700 cfs pumping capacity compared to 2,775 cfs for G-370 (originally designed for 2,175 cfs). No modifications in conveyance capacity are anticipated at this time.

4. CANAL CAPACITY ENHANCEMENT

4.1 North New River Canal Modifications

Three alternatives were considered:

- No modification, operate the canal at full capacity with no velocity restrictions.
- No modification, reduce capacity to restrict velocity to less than 2.5 fps.
- Modify the canal to eliminate choke points and allow full capacity with velocities less than 2.5 fps.

Table 1 represents the available capacity for the first alternative.

Table 2 provides the capacity (obtained from HEC-RAS modeling) of the NNR Canal if the velocities were limited to 2.5 fps.

For the third alternative, cross sections were developed to establish the amount of canal expansion required to result in the capacities shown in Table 1 maintaining a maximum velocity of 2.5 fps.

HEC-RAS modeling of the existing canal with two pumps operating at G-370 indicated maximum velocities between 2.5 and 3.3 feet per second (fps) at river sections 17, 18, and 19. An increase in area from 20% to 30% at each of these sections reduced the velocities to below 2.5 fps. The excavations required to reduce the velocity range from 10 to 20 feet horizontally into the northeast bank of the canal.

HEC-RAS modeling of the existing canal with three pumps operating at G-370 indicated maximum velocities between 2.5 and 3.0 feet per second (fps) at river sections 8, 9, and 17. An increase in area from 10% to 30% at each of these sections reduced the velocities to below 2.5 fps. The excavations required to reduce the velocity range from 10 to 20 feet horizontally into the northeast bank of the canal.

4.1.1 North New River Canal Excavation Quantities and Materials

The excavations required to expand the NNR Canal in order to limit velocity to 2.5 fps will include four types of material. These materials include the upper muck and existing excavated canal material layer, caprock, silty sand with gravel layer, and sediment within the canal. The following assumptions were made for the estimation of preliminary quantities of excavated materials:

- Maximum flows used in these calculations do not include additional flows from Bolles & Cross Canals.
- Depth to caprock along canal is an average of approximately 3.5 feet based on borings taken along the Reservoir A-1 northeast side in 2004 by Nodarse & Associates, Inc.
- Thickness of caprock along the canal is an average of 10 feet based on borings taken along the Reservoir A-1 northeast side in 2004 by Nodarse & Associates, Inc.
- All caprock excavation, silty sand with gravel, and sediment within the canal will be excavated under water.

Table 3 shows the quantities of the excavated materials.

Cross sections for each area (cross sections 8, 9, 17, 18, and 19) of the canal requiring modification are included in the appendix.

4.1.2 North New River Canal Modification Opinion of Costs

Based on the excavation quantities listed above, the preliminary cost to modify the NNR Canal in order to reduce maximum velocity below 2.5 fps is $5,590,000. A summary of the preliminary opinion of costs is included in Table 4. The opinion of costs includes mobilization, overhead and profit, a 30% contingency, and 5% for project reserve.

4.2 STA 3/4 Inflow Supply Canal

No modifications to the STA 3/4 Inflow Supply Canal are required except as may be required for pumping or inlet/outlet needs. These will be covered under a future technical memorandum.

4.3 Miami Canal

No modifications to the Miami Canal are anticipated at this time.

5. CANALS REQUIRED TO SERVE RESERVOIR A-1

Figure 4 indicates the three canals serving Reservoir A-1. These include the Northeast Pump Station Connector Canal, Seepage Canals, and Internal Perimeter Canal.

5.1 Northeast Pump Station Connector Canal

The connector canal will be constructed from the NNR Canal to the proposed Northeast Pump Station. The following assumptions were made in order to determine an appropriate cross section:

- Maximum Northeast Pump Station and connector canal capacity of 3,300 cubic feet per second (cfs).
- Maximum outflow from the reservoir is 2,500 to 3,000 cfs based on the irrigation demands as provided by the water balance modeling.
- Maximum length of canal from NNR Canal to pump station of 800 feet.
- Side slopes of 2H to 1V.
- Maximum velocity of 2 fps.

- Unlined earthen side slopes with an average Manning's n value of .030 for the entire perimeter.
- Maximum water surface elevation in NNR Canal at pump station of 9.13 NAVD 1988 at maximum connector canal flow of 3,300 cfs. Elevation is based on HEC-RAS modeling of NNR Canal.
- Maximum water surface elevation in NNR Canal at pump station of 9.85 NAVD 1988 at connector canal flow of 1,775 cfs (based on 3 pumps operating at G-370). Elevation is based on HEC-RAS modeling of NNR Canal.
- Bottom of connector canal set at -11.4 NAVD 1988 (approximately 2 feet above bottom of NNR Canal at pump station.

A hydraulic analysis of the connector canal indicated that a canal with a bottom width of 40 feet would produce a velocity of less than 2 feet per second and would result in a hydraulic drawdown of less than .05 feet from the NNR Canal to the pump station. Figure 5 shows a typical cross section through the connector canal. It will be necessary to construct berms above existing grade on either side of the canal near the pump station as the surrounding grade of approximately 8.0 NAVD 1988 is below the maximum canal water level of 9.85 NAVD 1988. The berms can also serve as maintenance and access road. The top of the berms will be at elevation 13.2 NAVD 1988.

5.1.1 Northeast Pump Station Canal Excavation Quantities

The excavations required to construct the Northeast Pump Station Connector Canal will include three types of material including the upper muck material layer, caprock layer, and silty sand with gravel layer. The following assumptions were made for the estimation of preliminary quantities of excavated materials:

- Depth to caprock along canal is an average of approximately 2 feet based on borings located near the Northeast Pump Station in 2004 by Nodarse & Associates, Inc.
- Thickness of caprock along the canal is an average of 7 feet based on borings along the Reservoir A-1 northeast side in 2004 by Nodarse & Associates, Inc.

Table 5 shows the quantities of the excavated materials.

5.1.2 Northeast Pump Station Connector Canal Opinion of Costs

Based on the excavation quantities listed above, the preliminary cost to construct the Northeast Pump Station Connector Canal. The cost includes the cost of a bridge over the canal for Highway 25. A summary of the preliminary opinion of costs is included in Table 6. The opinion includes mobilization, overhead and profit, a 30% contingency, and 5% for project reserve.

5.2 Seepage Canals

The seepage canals around the exterior of Reservoir A-1 will transport seepage from the reservoir to seepage pumps located at the Northeast and G-370 pump stations. The canal dimensions indicated below were determined based on obtaining material for the construction of the reservoir embankment. The check of the hydraulics is based on the following assumptions for the seepage canal:

- 20 foot wide bottom, 2H to 1V side slopes, and 10 foot average canal depth.
- Unlined earthen side slopes with average Manning's n value of .030.
- Minimum of 2 feet of freeboard.
- Flow rate of 136 cfs (from reservoir seepage calculations).
- Length of longest section of seepage canal of 41,000 feet.
- For Manning's equation calculation, seepage canal is divided into 10 sections with equal seepage inflow of 13.6 cfs.

Based on the analysis of the seepage canals, hydraulic headloss and velocities will not be a concern. Table 7 presents results (velocity and hydraulic drawdown) for various depths of water in the seepage canal.

5.2.1 Connection from Seepage Canals to NNR Canal

The connection of seepage canals to the NNR Canal is not recommended for several reasons including:

- If the canals were connected the seepage canal water level is going to match the level in the NNR Canal. In order to provide the same freeboard it would require a berm approximately 5 to 6 feet high along the seepage canals.
- During high water periods in the NNR Canal, seepage may not be able to be adequately removed in which case gates and pumping may be required.

5.3 Supply and STA 3/4 Seepage Canals

The existing seepage canal located on the north side of the Supply and STA 3/4 canals convey seepage to pump station G-372 to the west and G-370 to the east. The 2004 Operation Plan for Stormwater Treatment Area-3/4 indicates that each pump station has a firm capacity of 150 cfs. The seepage canal has a 10 foot wide bottom with 2H to 1V side slopes with a water surface depth range from 6.5 to 8.0 feet. Water in the canal is split with approximately half going to G-372 and half to G-370 (assuming the same number of pumps are operating at each pump station). Deleting the north seepage canal along the south and southwest side of the proposed Reservoir A-1 will eliminate approximately 9 miles of the 16.4 mile distance from G-372 to G-370. Thus more than half of the seepage will be eliminated. Since both pump stations have the same capacity, G-372 (after Reservoir A-1 is constructed) will have sufficient pumping capacity to handle the flow in the remaining seepage canal located north of the Supply Canal.

The existing seepage canal located north of the Supply Canal will be connected (by a gate structure) with the proposed seepage canal located on the west and north sides of the proposed Reservoir A-1 so that they can operate together or independently.

5.4 Internal Perimeter Canal

The primary function of this canal is to provide material for the zoned embankment option and the dimensions will be based accordingly. In the event that an RCC embankment is constructed, there will be no need for this material and the internal perimeter canal would not be constructed.

However, should the embankment be an earthen embankment, sufficient material would be required that the internal canal would extend around the entire perimeter of the reservoir. The internal perimeter canal will interconnect with existing agricultural canals within the reservoir and therefore improve drainage from the reservoir during low water levels. To some degree, the internal perimeter canals will address deep water refugia requested by Fish and Wildlife. The perimeter canal and existing agricultural canals would result in about 5 percent of the reservoir floor as deep water refugia.

6. EXISTING CANAL IMPACTS

The agricultural canals immediately to the west of Reservoir A-1 are currently drained or irrigated from the North New River Canal. Refer to Figure 1. The area in question is the cross hatched area between the drainage area boundary and the A-1 Reservoir. A method will need to be developed to drain and irrigate this area after Reservoir A-1 is constructed and before Reservoir A-2 is constructed. Options that could address irrigation and drainage of this area include:

- Realignment of the drainage boundary and such physical modifications to provide drainage/irrigation service from the Miami Canal.
- Modification to convey drainage/irrigation water to and from the STA 3/4 Supply Canal. Gates and pumps would be required.
- Modifications to convey drainage/irrigation water via the A-1 Reservoir seepage canal. If the seepage canal is allowed to be connected to the NNR Canal, then it could be used to provide irrigation to and drain water from the agricultural area.

The method used to supply the agricultural canals will be finalized during the preliminary design.

The cross sections on Figures X through X show the existing canal, above water excavation of muck and canal material, below water caprock excavation, and below water silty sand and gravel excavation. All cross sections are looking towards the southeast (Excavations are on the northeast side of the canal).

7. REFERENCES

2004 Reservoir A-1 Soil Borings by Nodarse & Associates, Inc.

SFWMD HEC-RAS canal models.

Partial Definite Project Report – Central and Southern Florida Project – For Flood Control and Other Purposes – Part I – Agricultural and Conservation Areas – Supplement 13, Design Memorandum, Hydrology and Hydraulic Design of North New River Canal and Related Works (L-18, L-19, L-20, and S-7), Corps of Engineers, U.S. Army, Office of the District Engineer, Jacksonville, FLA., July 6, 1953, p 10.

Operation Plan – Stormwater Treatment Area – 3/4, May 2004, South Florida Water Management District.

TABLES

Table 1 – North New River Canal Current Capacity with No Velocity Restrictions			
Condition	**G-370 Output (cfs)**	**NE P.S. Output (cfs)**	**Total Flow to Reservoir A-1 (cfs)**
Local Rainfall (3/4”)	2,775	1,525-1,775	4,300-4,550
Local Rainfall (3/4”)	1,850	3,000-3,300	4,850-5,150
No Rainfall	2,775	-	2,775
No Rainfall	1,850	1,375	3,225

Table 2 – North New River Canal Capacity with Maximum Velocity of 2.5 fps			
Condition	**G-370 Output (cfs)**	**NE P.S. Output (cfs)**	**Total Flow to Reservoir A-1 (cfs)**
Local Rainfall (3/4”)	2,350	1,575	3,925
Local Rainfall (3/4”)	1,850	1,960	3,810
No Rainfall	2,350	525	2,875
No Rainfall	1,850	1,260	3,110

Table 3 – North New River Canal Excavation	
Excavated Material	**Quantity (cubic yards)**
Soil excavated above water level	10,000
Caprock excavated below water level	90,000
Silty sand, gravel and sediment below water level	130,000

Table 4 – North New River Canal Modification Opinion of Costs	
Muck	$ 30,000
Caprock	$ 4,840,000
Silty Sand and Gravel	$ 720,000
Total Cost	**$ 5,590,000**

Table 5 – Northeast Pump Station Connector Canal Excavation	
Excavated Material	**Quantity (cubic yards)**
Muck and soil excavation	15,000
Caprock excavation	20,000
Silty sand and gravel excavation	19,000

Table 6 – NE Pump Station Connector Canal Opinion of Costs	
Muck	$ 50,000
Caprock	$ 1,080,000
Silty Sand and Gravel	$ 100,000
Highway 25 Bridge Over Canal	$ 10,000,000
Total Cost	**$ 11,230,000**

Table 7 – Seepage Canal Hydraulics (136 cfs)		
Average water depth (ft)	Velocity at Pump Station (fps)	Drawdown at Pump Sta. (ft)
8.0	.47	.16
7.5	.52	.21
7.0	.57	.27
6.5	.63	.36

FIGURES

Figure 1 – Reservoir and Canal Location

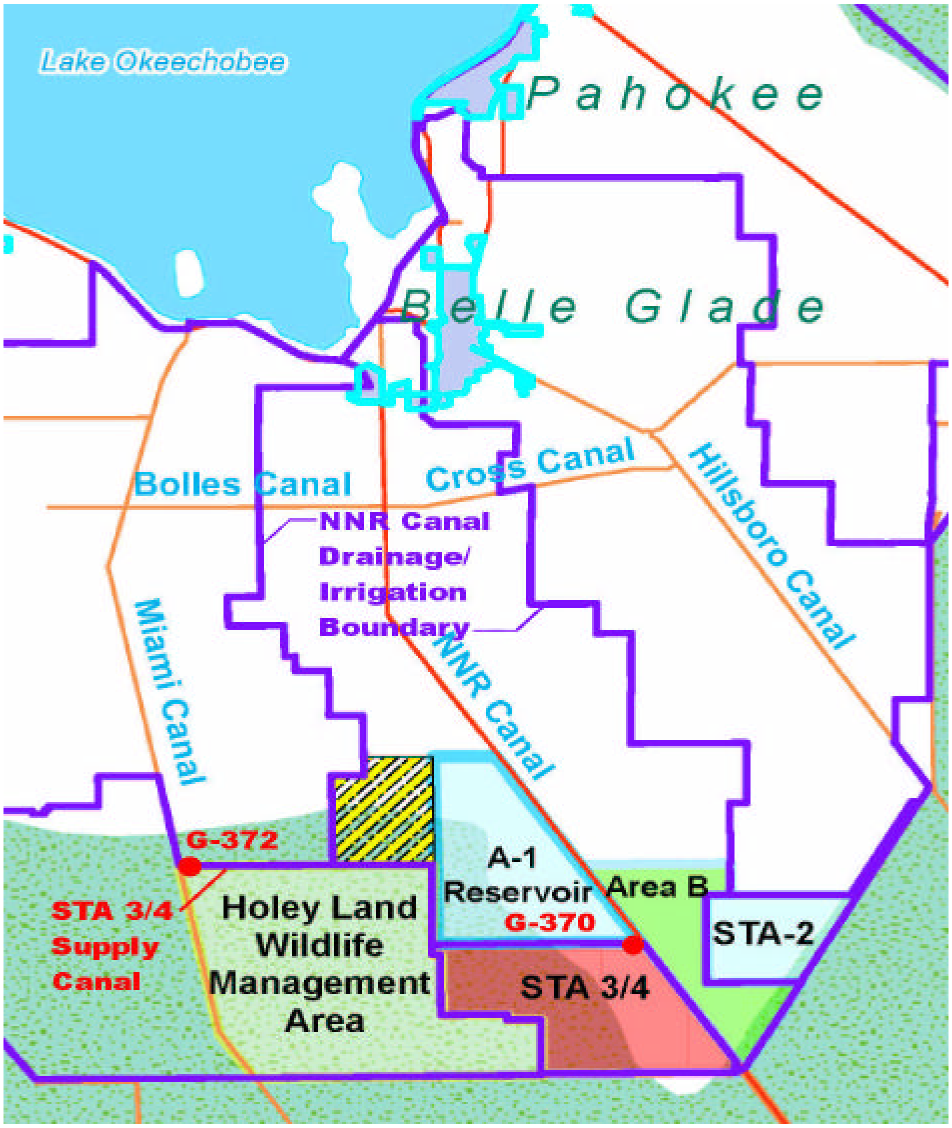

Figure 2 – North New River Canal Profile (Elevation in NGVD)

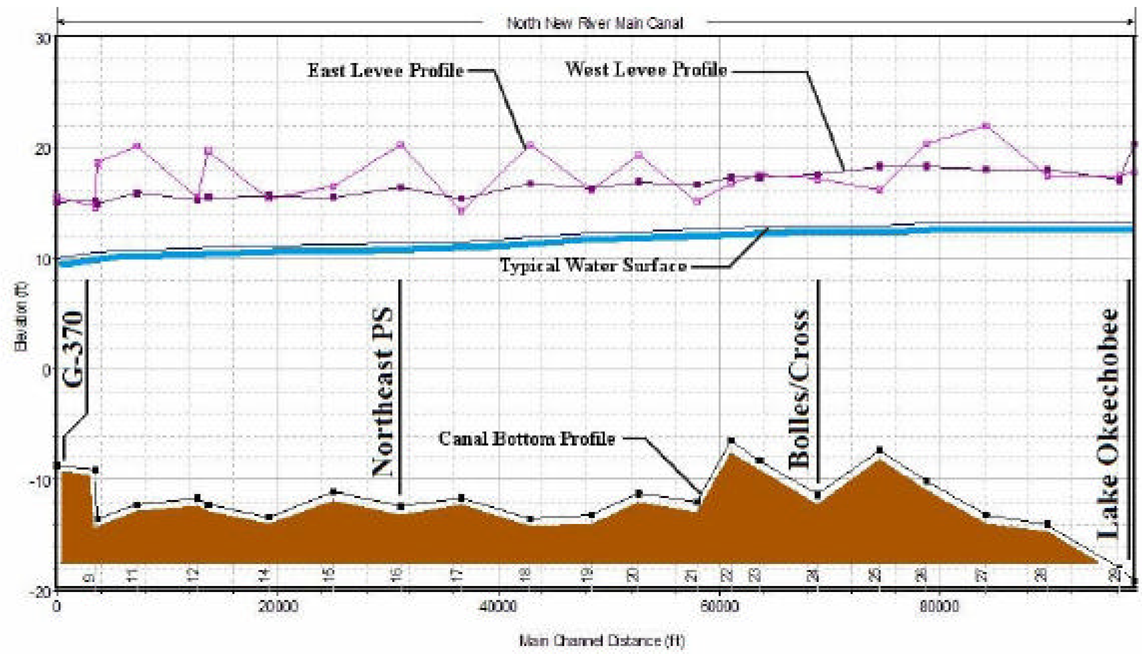

Figure 3 – North New River Canal Velocity with Potential Restrictions (Elevation in NGVD)

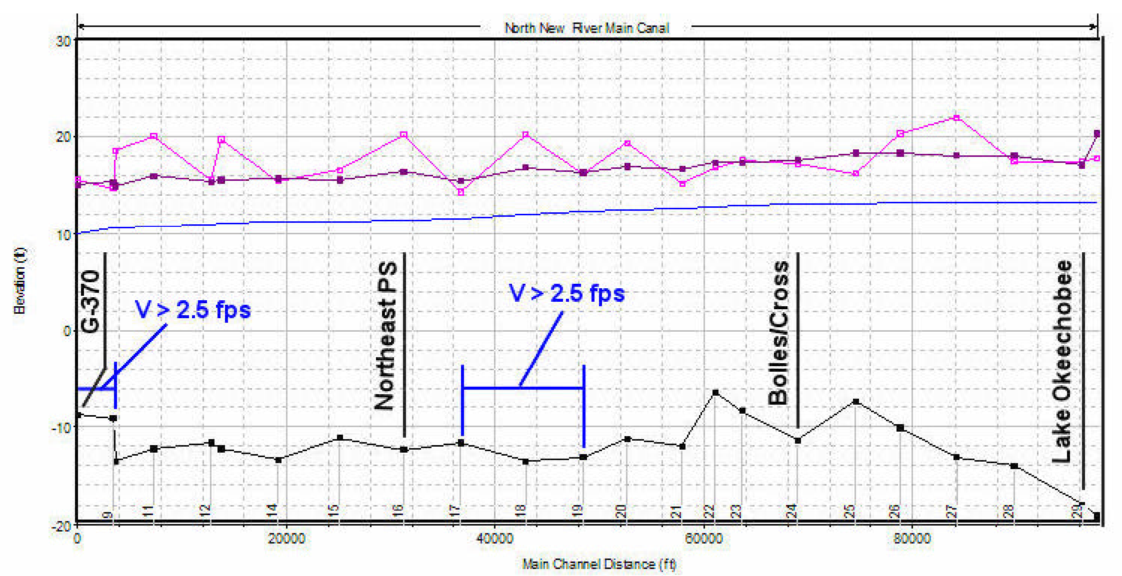

Figure 4 – Canals Serving Reservoir A-1

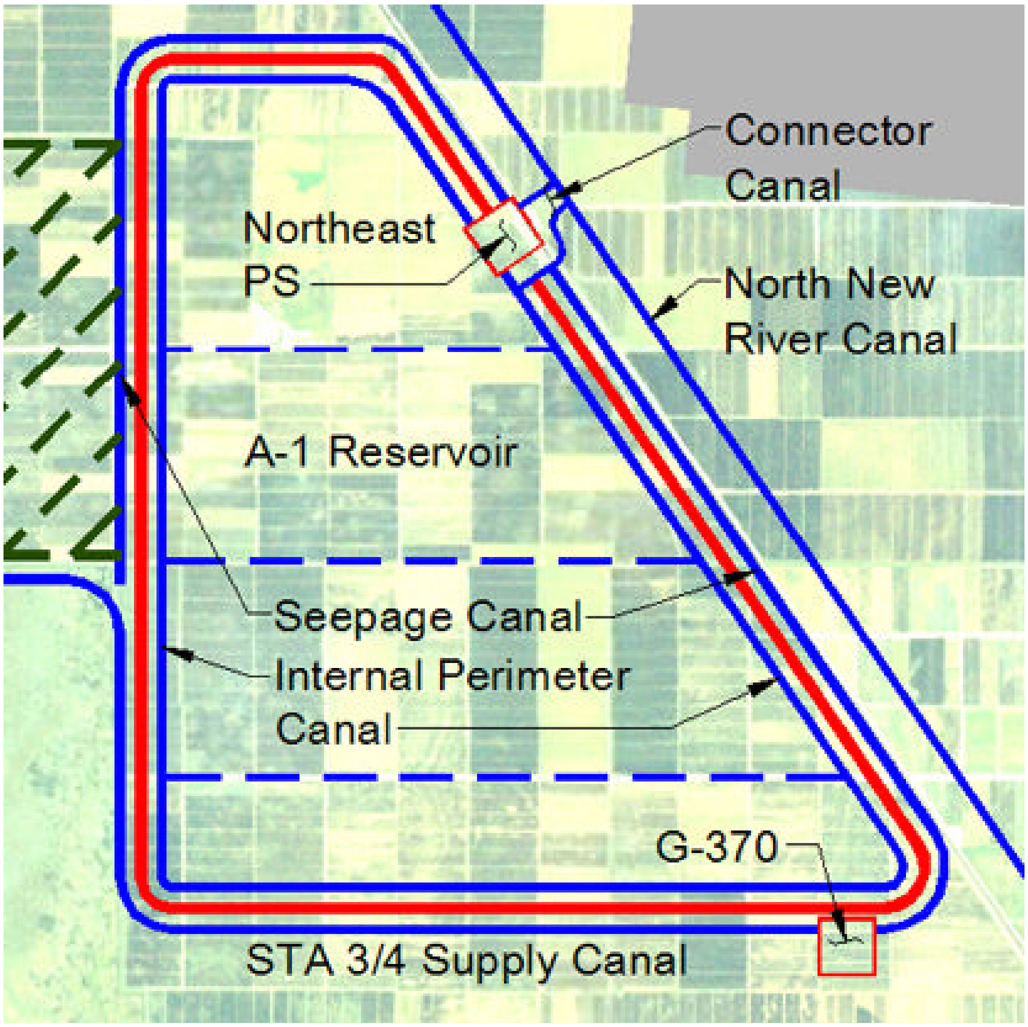

Figure 5 – Typical Northeast Pump Station Connector Canal Cross Section

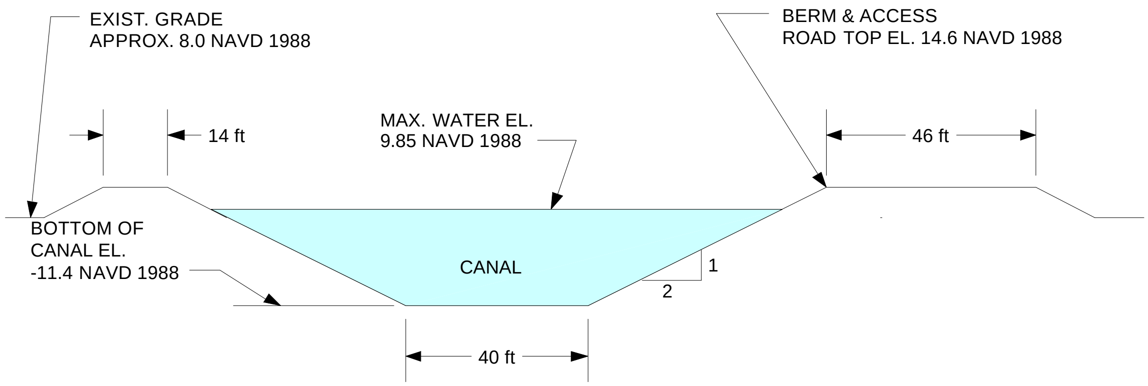

Figure 6 - Cross Section No. 8

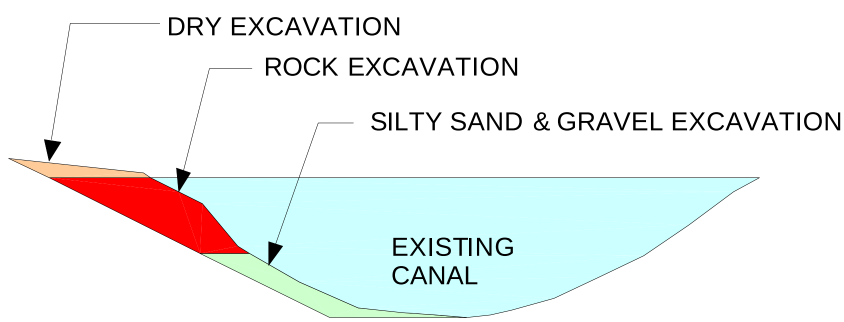

Figure 7 - Cross Section No. 9

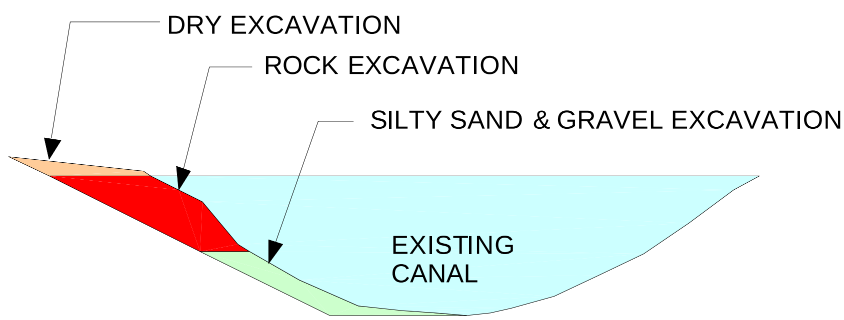

Figure 8 - Cross Section No. 17

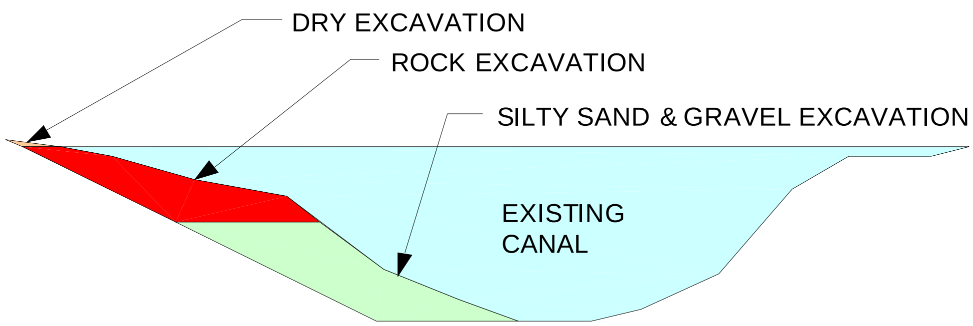

Figure 9 – Cross Section No. 18

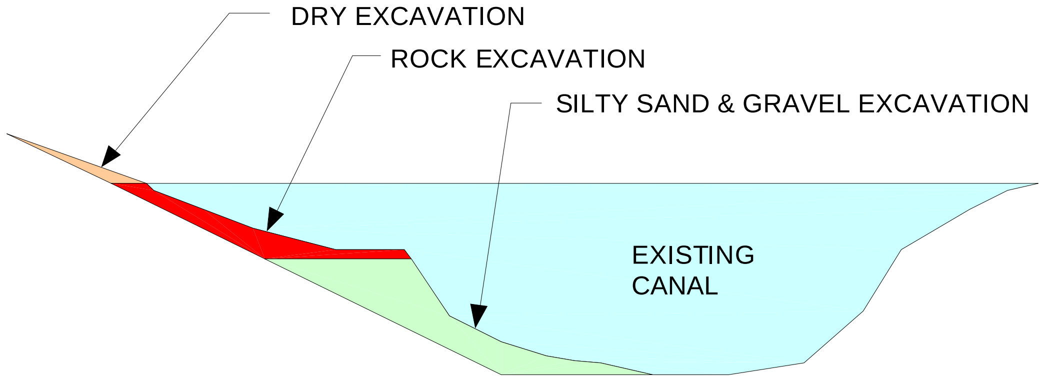

Figure 10 – Cross Section No. 19